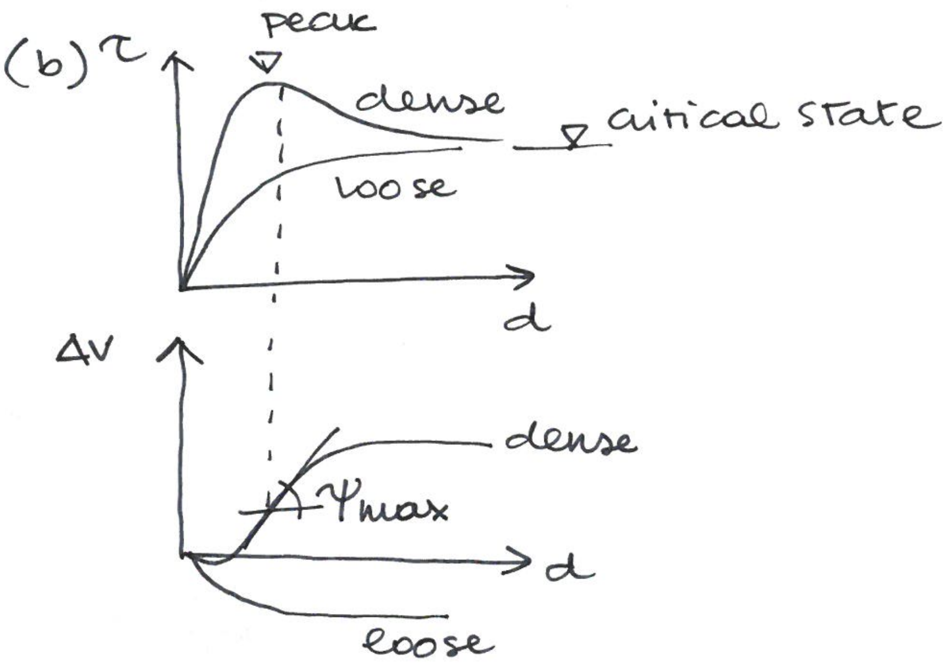
1 (a) $\gamma_d = \frac{G_s}{1+e}\gamma_w = \frac{2.7 \times 10}{1.8} = 15.0 \text{ kN/m}^3$

$$\gamma_{sat} = \frac{G_s + e}{1+e}\gamma_w = \frac{3.5 \times 10}{1.8} = 19.4 \text{ kN/m}^3$$

(b)



dense sands dilate, so extra work is required for shearing before critical state (shear at constant volume) is reached. At critical state soil will mobilise φ_{cs}. φ_{peak} depends on stress level. loose sands contract throughout and have no peak strength.

φ_{cs} is a reliable value to be used in design. Adopting φ_{peak} may lead to instability under cyclic loading due to progressive failure

(c) from databook

$\sigma = \gamma z \cos^2 \beta$

$u = \gamma_w z_w \cos^2 \beta$

- $\sigma' = \sigma - u = (\gamma z - \gamma_w z_w) \cos^2 \beta$
- $\tau = \gamma z \cos \beta \sin \beta$
- $\gamma = \dfrac{\gamma_d (z - z_w) + \gamma_{sat} z_w}{z}$
- $\varphi_{mob} = \tan^{-1}\left(\dfrac{\tau}{\sigma'}\right)$

DRY SEASON

$z_w = 2m$

$\gamma = \dfrac{15 \times 4 + 19.4 \times 2}{6} = 16.5 \text{ kN/m}^3$

$\sigma' = (16.5 \times 6 - 10 \times 2) \times \cos 25° = 81.2 \text{ kPa}$

$\tau = 6 \times 16.5 \times \cos 25° \times \sin 25° = 37.9 \text{ kPa}$

$\varphi_{mob} = \tan^{-1}\left(\dfrac{37.9}{81.2}\right) = 30.3°$

$\varphi_{mob} < \varphi_{cs}$ the slope is stablee

$SF = \dfrac{tg\, \varphi_{cs}}{\tan \varphi_{mob}} = \dfrac{0.70}{0.58} = 1.21$ ✓

WET SEASON

$z_w = 4m$

$\gamma = \dfrac{15 \times 2 + 19.4 \times 4}{6} = 18 \text{ kN/m}^3$

$\sigma' = (18 \times 6 - 10 \times 4) \times \cos 25° = 55.7 \text{ kPa}$

$\tau = 18 \times 6 \times \cos 25° \times \sin 25° = 41.3 \text{ kPa}$

$\varphi_{mob} = \tan^{-1}\left(\dfrac{41.3}{55.7}\right) = 36.6°$

$\varphi_{mob} > \varphi_{cs}$ the slope may fail

however

the sand will have a peak strength depending on relative density and stress

$$I_D = \frac{e_{max}-e}{e_{max}-e_{min}} = \frac{0{,}95-0{,}8}{0{,}95-0{,}6} = 0{,}43$$

assuming $k_0 \simeq 0{,}5$ and $p' \simeq \sigma'\left(\frac{1+2k_0}{3}\right) = \frac{2}{3}\sigma'$

$$p' = \frac{2}{3}\,55{,}7 = 37{,}1 \text{ kPa}$$

$$I_C = \ln\left(\frac{\sigma_c}{p'}\right)$$

σ_c material constant
if we assume (for safety) carbonate sand

$\sigma_c = 5000$ kPa

$$I_C = \ln\left(\frac{5000}{37{,}1}\right) = 4{,}90$$

$$I_R = I_D \cdot I_C - 1 = 0{,}43 \times 4{,}90 - 1 = 1{,}1$$

(c) $\Delta\varphi = \varphi_{peak} - \varphi_{cs} = 5 \times I_R = 5{,}5°$

$$\varphi_{peak} = \varphi_{cs} + \Delta\varphi = 35 + 5{,}5 = 40{,}5°$$

$\varphi_{mob} < \varphi_{peak}$

the slope may not fail

$$(FS)_{peak} = \frac{tg\ 40{,}5°}{tg\ 36{,}6°} = 1{,}15$$

(d)

z_w (m)	6	5	5.5
γ (kN/m^3)	19.4	18.7	19.1
σ' (kPa)	46.5	51.1	48.8
τ (kPa)	44.7	43.0	43.8
φ_{mob} (°)	43.8°	40.1°	41.9°
p' (kPa)	31	34.1	32.6
I_C (-)	5.08	4.99	5.03
I_R (-)	1.18	1.14	1.16
$\Delta\varphi$ (°)	5.9	5.70	5.80
φ_{peak} (°)	40.9°	40.7	40.80
	fail	ok	fail

the slope will fail for a water table

$z_w = 5 \div 5.5$ m

(c) plot $\sigma_c - \sigma_0$ versus $\ln \gamma_c$
where $\sigma_0 = 490$ kPa and $\gamma_c = 2\epsilon_c/100$

$\sigma_c - \sigma_0$ (kPa)	γ_c	$\ln \gamma_c$
110	0.0014	-6.57
260	0.0076	-4.88
410	0.0170	-4.07
560	0.0294	-3.53
630	0.0453	-3.10
635	0.0459	-3.08
725	0.0715	-2.64
790	0.1005	-2.30
860	0.1468	-1.92

find cu as gradient of plot between 860 kPa and 410 kPa (first two points discarded) (see plot)

$$c_u = \frac{860-410}{4.07-1.92} = 212 \text{ kPa} \simeq 200 \text{ kPa}$$

(d) @ $\sigma_c = 1350$ kPa

$$\sigma_\theta = \sigma_c - 2c_u = 1350 - 400 = 950 \text{ kPa}$$

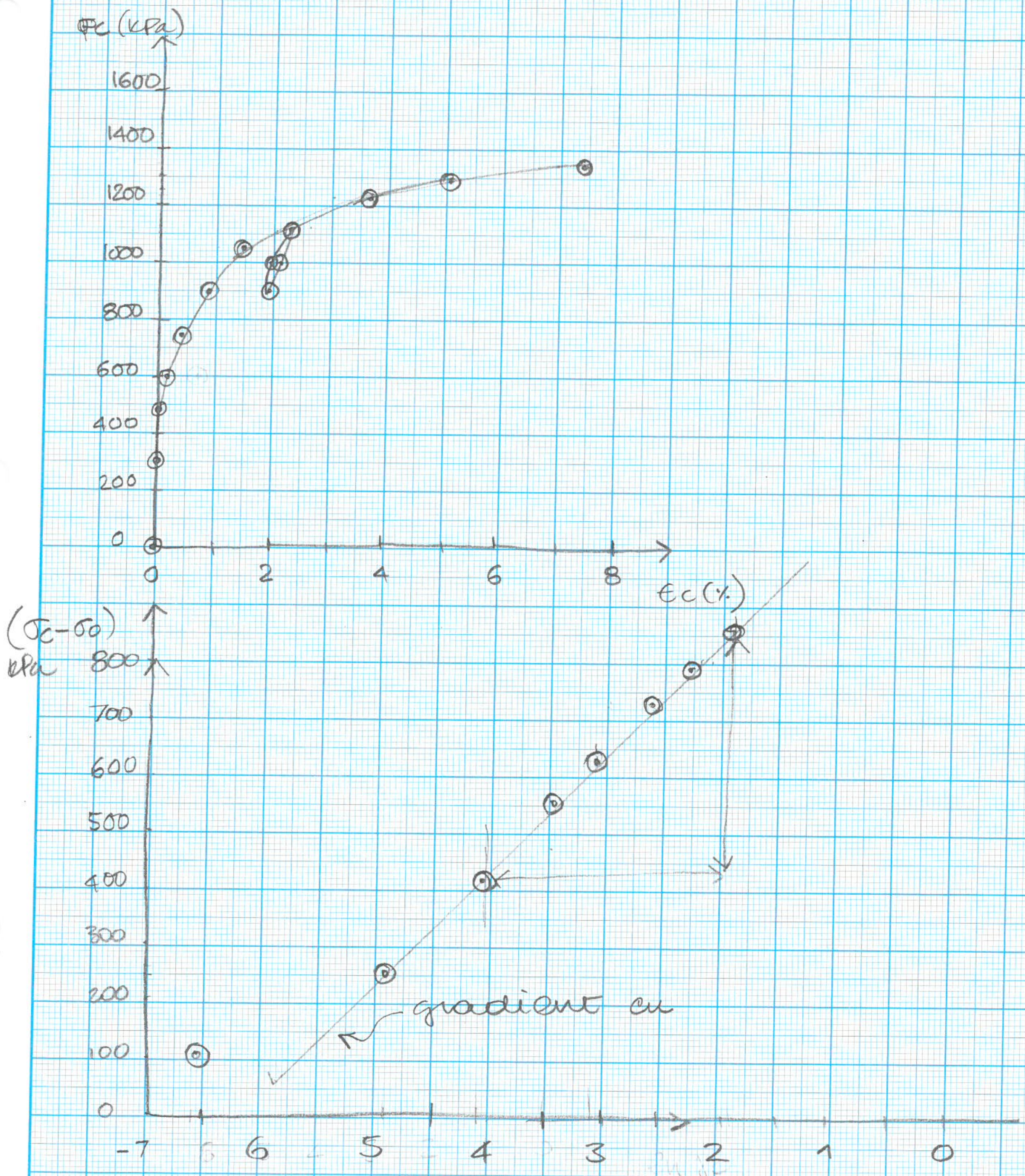
$$p = \tfrac{1}{2}(\sigma_c + \sigma_\theta) = \tfrac{1}{2}(1350+950) = 1150 \text{ kPa}$$

$$R = p' \sin\varphi' \qquad p' = \frac{R}{\sin\varphi'} = \frac{c_u}{\sin\varphi'} = \frac{200}{\sin 23^\circ} = 512 \text{ kPa}$$

$$u = p - p' = 1150 - 512 = 638 \text{ kPa}$$

or $$\Delta u = c_u \ln\left[\frac{G}{c_u} \cdot \frac{\Delta V}{V}\right] = c_u \ln\left[\frac{G}{c_u} \cdot 2\epsilon_c\right]$$

$$= 200 \times \ln\left[\frac{32352}{200} \times \frac{7.34 \times 2}{100}\right] = 634 \text{ kPa}$$



σc (kPa)
1600
1400
1200
1000
800
600
400
200
0
0
2
4
6
8
εc (%)
(σc−σo) kPa
800
700
600
500
400
300
200
100
0
-7
6
5
4
3
2
1
0
gradient cu

2 (a) $\sigma_{ho} \simeq 490$ kPa

$\sigma_{vo} = 24 \times 19 = 456$ kPa

$u = 22 \times 10 = 220$ kPa

$\sigma'_{ho} = \sigma_{ho} - u = 270$ kPa

$\sigma'_{vo} = \sigma_{vo} - u = 236$ kPa

$$k_o = \frac{\sigma'_{ho}}{\sigma'_{vo}} = \frac{270}{236} = 1.14$$

$$k_o = k_{onc} \cdot \sqrt{OCR} = (1-\sin\varphi'_{cs})\sqrt{OCR} \Rightarrow OCR = \left(\frac{k_o}{1-\sin\varphi'_{cs}}\right)^2$$

$$OCR = \left(\frac{1.14}{1-\sin 23^\circ}\right)^2 = 3.61$$

overconsolidated clay

(b) G_i from gradient between 750 kPa and 600 kPa:

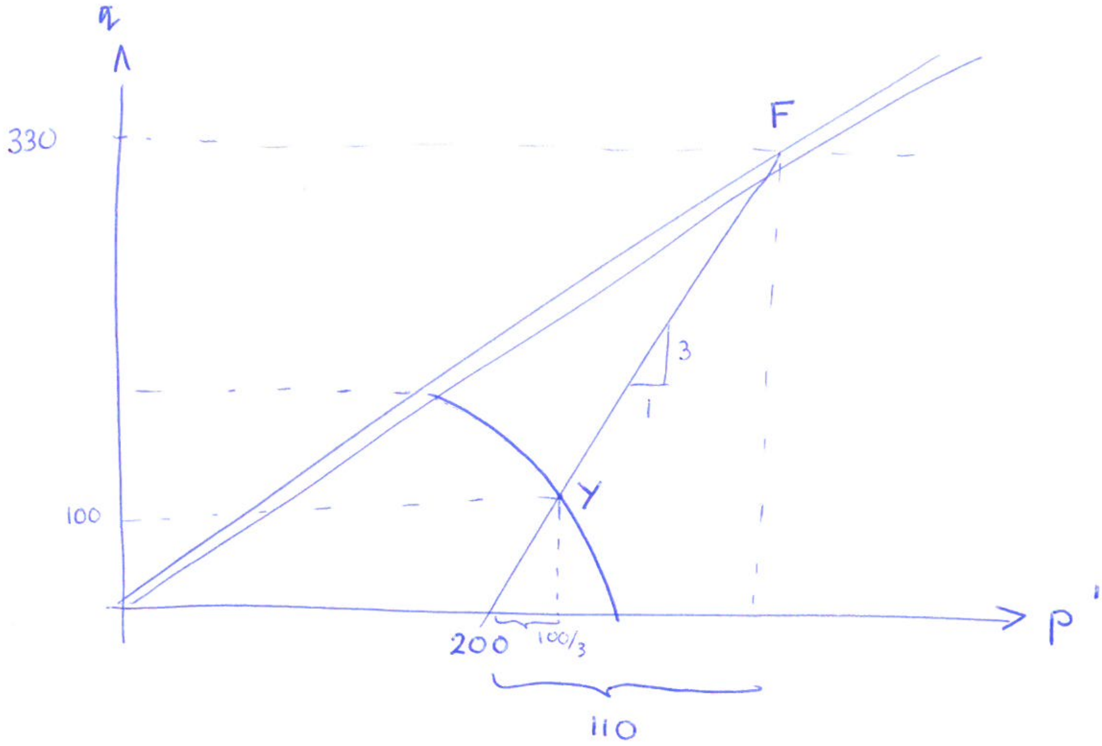
$$G_i = \frac{1}{2}\frac{\Delta\sigma_c}{\Delta\epsilon_c} = \frac{1}{2}\frac{750-600}{0,38-0,07} \times 100 = 24194 \text{ kPa}$$

G_{ur} from gradient of unload-reload loop:

$$G_{ur} = \frac{1}{2}\frac{\Delta\sigma_c}{\Delta\epsilon_c} = \frac{1}{2}\frac{1120-900}{2,26-1,92} \times 100 = 32352 \text{ kPa}$$

this last value is larger than G_i because free from disturbance errors, hence more credible

3.



$$M = \frac{330}{310} = 1.06$$

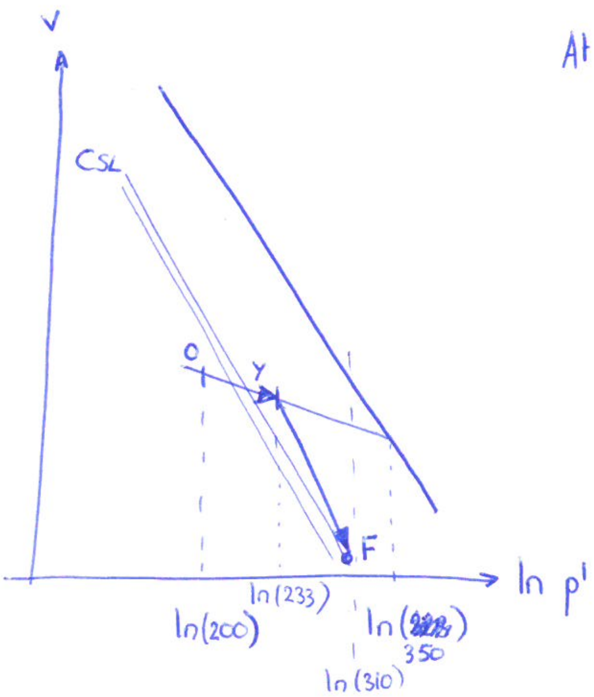
Yield surface

$$q = M p' \ln \frac{p'}{p'_c}$$

$$100 = 1.06 \times 233\tfrac{1}{3} \ln\left(\frac{\cancel{233\tfrac{1}{3}}\ p'_c}{p'_o}\right)$$

$$p'_c = \cancel{\ \ } = \underline{\underline{350 \text{ kPa}}}$$

b)



At O : $v = \Gamma^{+\lambda-\kappa} - \lambda \ln(350) + \kappa \ln\left(\frac{350}{200}\right)$

$= \cancel{1.938}\ 2.058$

Y : $v = \cancel{1.938}\ 2.058 - \kappa \ln\left(\frac{233}{200}\right)$

$= \cancel{1.933}\ 2.053$

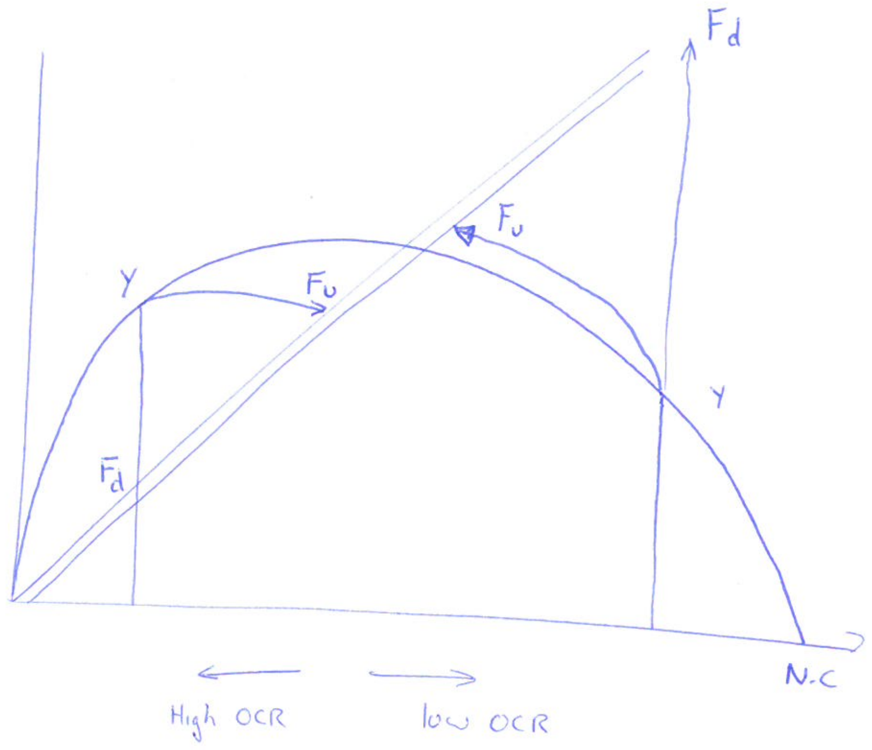
$\varepsilon_v = \underline{0.24\%}$

F : $v = \Gamma - \lambda \ln(310)$

$= 1.939$

$\varepsilon_v = \underline{\underline{5.8\%}}$

4.



High OCR soils yield above critical state and then soften to return to CSL at high strain. In undrained loading they generate negative pore pressures reducing the amount of softening occurring

Low OCR soils yield below c.s. and harden to get to CSL, especially when drained. In undrained conditions they generate +ve e.p.p. and hence have lower strength than drained.

b) Drained:

$$\tau_{c.s.} = \sigma'_o \tan \phi_{crit}$$

$$\tau_y = \sigma'_o \tan \phi_{crit} \ln \left(\frac{\sigma'_{max}}{\sigma'_o} \right)$$

$$= \sigma'_o \tan \phi_{crit} \ln (OCR)$$

Undrained

$$\tau_y = \sigma'_o \tan \phi_{crit} \ln (OCR) \quad [\text{as before}]$$

$$V = \Gamma + \lambda - K - \lambda \ln \sigma'_{max} + K \ln (OCR)$$

at c.s. $\quad V = \Gamma - \lambda \ln \sigma'_f$

Equating V $\quad \sigma'_f = \exp^{\left(\frac{K}{\lambda} - 1\right)} \sigma'_{max} \, OCR^{-K/\lambda}$

$$\tau_{c.s.} = \sigma'_f \tan\phi$$

$$= \sigma'_0 \, e^{(\kappa/\lambda - 1)} \, OCR^{(1-\kappa/\lambda)} \tan\phi_{crit}$$

c) Slope stability - governed by lowest strength.

OCR > e drained strength governs but there may be substantial peak strength. Slopes steeper than ϕ_{crit} may be stable short term but may suffer long-term creeping failure.

N.C soils fail undrained and cannot stand at ϕ_{crit} owing to generation of pore-pressure.

Construction equipment:

need to overcome peak strength, but only in short term.

High OCR soils yield strength is important + potentially suction generation.

Low OCR soils - don't need to consider drained strength as excess p.p. will help.